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Performance Monitoring for Sustainable Concrete Infrastructure

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ABSTRACT

Satisfactory guidelines for ensuring adequate reinforced concrete durability can only be developed by monitoring concrete performance under a range of field exposure conditions over an extended period of time. Only then can there be a move from prescriptive durability specifications (minimum grade, maximum water-binder ratio, and minimum cement content) to performance-based methods. The situation is also made more complex by the range of cements now available - BS EN 197 defines a total of 27 products in the family of common cements. Implementation of both design for durability and performance-based standards and specifications are limited by the lack of rapid, simple, science-based test methods for characterizing the transport properties and deterioration resistance of concrete. This paper presents an overview of performance-based specification and developments in the application of electrical property measurements as a testing methodology in evaluating the relative performance of concrete mixes. The technique lends itself to in-situ monitoring thereby allowing measurements to be obtained on the *as-placed* concrete.

Keywords: concrete, durability, performance, resistivity, formation factor, testing.

1. INTRODUCTION

It is an inescapable fact that our infrastructure deteriorates with time. A state of the nation report [1] graded the quality of the UK's infrastructure from B (fair) to D (poor) with an overall grading of C (average). The picture is the same in the US, where the ASCE (1998) Report on America's Infrastructure estimated a five-year total investment need of US\$1.3 trillion just to reinstate roads, bridges and other infrastructure systems to good serviceable life; in 2005, this investment need has risen to US\$1.6 trillion. Further to this, the average state of America's infrastructure was given a Grade D (Poor); as a specific example, in excess of 40% of 500,000 highway bridges are rated as structurally deficient or functionally obsolete and some US\$100 billion is the estimated requirement to eliminate the current backlog of bridge deficiencies and maintain repair levels [2].

Considering reinforced concrete structures, the initiation and propagation stages of deterioration result from a complex interaction of physical, chemical and electrochemical phenomena. The rate at which a particular structure deteriorates depends on many factors and evaluating the performance of reinforced concrete requires numerous data inputs, in particular, the response of the concrete to the changing micro-environment in the vicinity of a specific structural element or part of a structure. As it is the concrete cover-zone which protects the steel reinforcement - and therefore provides the first line of defence against the environment - it is understandable that considerable attention is directed towards assessing the performance of this zone [3]. The protective properties of the cover-zone are a major factor with regard to the in-service performance of the structure, likely deterioration rates for a particular exposure condition and compliance with specified design life. It is permeation processes such as diffusivity and sorptivity which are important and there clearly exists a need to study and determine quantitatively those near-surface characteristics of concrete which promote the ingress of gases or liquids containing dissolved contaminants.

Regarding concrete durability, European Standard BS EN206 [4] deals with durability on the basis of a prescriptive, deem-to-satisfy approach by specifying minimum grade, minimum binder content and maximum water-binder ratio for a series of well-defined environmental classes. Interestingly, section 5.3.3 of this code allows for performance-related methods and defines concrete on the basis of an *equivalent durability procedure* (EDP); further detail on the EDP is presented in PD CEN/TR 16563 [5]. However, in order to fully implement a performance-based approach requires,

- (i) long-term experience of local materials and practices and on a detailed knowledge of the local environment;
- (ii) test methods based on approved and proven tests that are representative of actual conditions and have approved performance criteria; and,
- (iii) analytical models that have been calibrated against test-data representative of actual conditions in practice.

2.0 PERFORMANCE TESTING

Although attempts have been made to introduce performance-based specifications, the lack of reliable, consistent and standardised test procedures for evaluating concrete performance has hindered its widespread implementation. Furthermore, there is also an intense need to evaluate the concrete as early as possible; the sooner information is obtained about the early-age properties of any given batch of concrete, the sooner adjustments can be made to the materials, proportions or

processes for subsequent concrete placement, and the sooner remedial measures can be initiated on the concrete already installed or construction practices altered (e.g. extended curing). Early-age testing is useful in this regard and, indeed, absolutely essential as the consequences of unsatisfactory concrete discovered at a later stage becomes expensive. The term *identity testing* is used in BS 8500-1:2015 [6] to describe testing to validate the identity of the mix. Identity testing attempts to verify some key characteristic of the concrete that relates to the desired performance and could take the form of a slump, flow, density, strength, water-content or some non-destructive or in-place method.

According to BS EN1990 [7], a reinforced concrete structure is designed in such a way that *'deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance'*. It is evident that consideration of a maintenance strategy forms an integral part of the structural design process. Implicit within this process is a consideration of adequate monitoring procedures, together with an appropriate level of inspection for the particular structure. As noted above, cover-zone transport mechanisms, their variation through the cover-zone, cyclic wetting/drying characteristics and temporal changes in properties all influence reinforced concrete performance and durability. In-situ monitoring of cover-zone concrete – in both the spatial and temporal domains - could thus assist in making realistic predictions as to the in-service performance of the structure; likely deterioration rates for a particular exposure condition, compliance with the specified design life and as an early warning indicator of incipient problems. Fig. 1 highlights the usefulness of in-situ monitoring of concrete performance during the life of the structure which can be divided into three stages: an initial stage which represents the concrete response to the external environment, a second stage which represents the initiation of the deterioration process (corrosion in this instance) and a third stage where propagation of deterioration occurs.

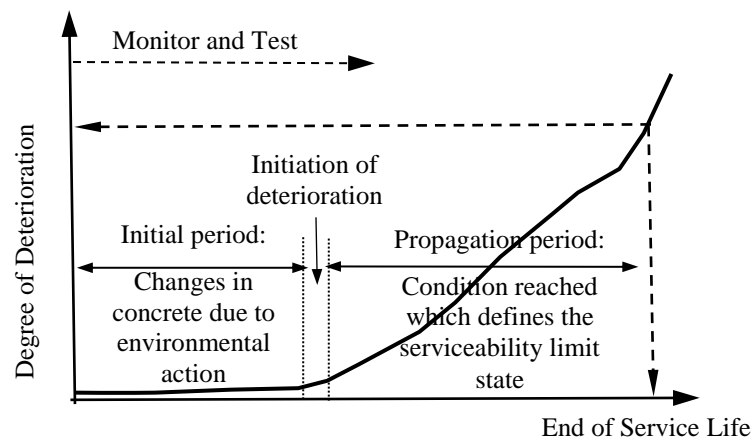


Fig. 1. Components of service life

The specification of the service-life/performance/durability of reinforced concrete by means a single parameter is the *holy grail* of the engineer and hence a subject of intense research interest. The challenges posed by a performance-based testing and specification approach to improve service life of concrete structures have been acknowledged for several years [8]. It is set against this background that this paper examines the use of the intrinsic electrical properties of cementitious materials (resistivity in the current work) as a potential candidate testing methodology in assessing concrete performance. The resistivity of concrete is a relatively easy parameter to evaluate and the measurement technology also lends itself to remote monitoring and interrogation. The installation of a network of sensors, embedded at critical locations within a concrete structure, could provide continuous, real-time data on the response of the micro-environment within the cover-zone to changing ambient environmental conditions. Furthermore, as the resistivity of the concrete

surrounding the rebar controls the magnitude of the corrosion current, it makes this parameter an important factor in the overall corrosion process.

3. ELECTRICAL PROPERTY MEASUREMENTS

From an electrical point of view, saturated concrete can be regarded as a composite comprising non-conductive aggregate particles embedded in an ionically conducting cementitious matrix, with conduction occurring primarily through the continuous pore-network. As a consequence, the electrical properties of concrete are directly related to pore tortuosity, pore connectivity and the degree of pore saturation, so its measurement could be of practical significance in assessing the durability of concrete structures. In addition, as the hydration process will alter the pore system beyond the standard curing period this will result in time-variant microstructural - hence electrical property - changes.

The electrical resistivity of a material is equal to the resistance, in ohms, between opposite faces of a unit cube of that material. If R is the bulk resistance of a prismatic sample of concrete placed between two electrodes of area A (cm^2), separated by a distance L (cm), then its bulk resistivity, ρ , is given by,

$$\rho = \left(\frac{RA}{L} \right) \text{ Ohm-cm } (\Omega\text{cm}) \quad (1)$$

Conventional treatment of resistivity data - saturated rocks in this instance - has been to utilise the formation factor F [9, 10] which is defined as the ratio of bulk resistivity, ρ , of the saturated rock to the resistivity of the saturating liquid, ρ_p . The formation factor is then related to the rock porosity, ϕ , through the relationship,

$$F = \frac{\rho}{\rho_p} = a\phi^{-m} \quad (2)$$

where the exponent m is the *cementation exponent*, and is related to the tortuosity and connectivity of the pore network within the rock and a is a correction factor which is valid over a particular range of porosities, ϕ . A wide range of values have been reported for m and a for different rock and sediment formations, with a , typically, in the range 0.4–2.5 and $m = 1.2$ –2.5 [11, 12]. Values of m and a are characteristic for a given porous rock formation and are determined empirically.

Provided that the solid phase can be regarded as an insulator in comparison to that of the interstitial aqueous phase, diffusion and resistivity of a saturated porous system are connected through the Nernst-Einstein relationship [13, 14],

$$\frac{D}{D_o} = \frac{\rho_p}{\rho} = \frac{1}{F} \quad (3)$$

where D is the diffusion coefficient of the porous material, D_o the free solution diffusion coefficient of the ion.

The work outlined above shows that the electrical resistivity of concrete provides an indirect measure of the capillary pore network (continuity, tortuosity) which is a key property in a concrete's resistance to the penetration of a fluid. In order to fully exploit electrical property measurements in assessing the relative performance of different concrete mixes, the resistivity of the pore-fluid, ρ_p , is

required or, at the very least, needs to be estimated. This is particularly relevant when supplementary cementitious materials (SCM) are used as these materials alter the concentrations of Na^+ , K^+ and OH^- within the interstitial aqueous phase in comparison to a plain Portland cement concrete. In order to evaluate the resistivity of the pore-fluid, pore expression techniques can be used; however, this is impractical for a variety of reasons, not least because it complicates the testing procedure. The pore-fluid resistivity could be estimated from the ionic concentrations predicted from the chemical composition of the binder. For example, the model of Taylor [15] predicts the concentration of various ionic species in the pore solution based on the cement composition and degree of hydration. From the estimated concentrations of the ions, it is then possible to evaluate the resistivity of the pore-fluid. In this work, the model developed by Snyder et al [16] and Bentz [17] is used to estimate the resistivity of the pore fluid.

4.0 EXPERIMENTAL DEVELOPMENTS

4.1 Cover-Zone Electrode Array

Electrical measurements were obtained utilizing a multi-electrode array [18] which was embedded within the cover-zone at the time of casting. This arrangement allowed monitoring of the spatial distribution of both electrical resistance and temperature. In summary, the array comprises six electrode-pairs and four thermistors mounted on a PVC former, with the former being secured onto two, 15mm diameter steel bars; the cover-to-steel was 50mm. With this arrangement, the electrode-pairs were positioned at 10, 20, 30, 40, 50 and 75mm from the concrete surface. The electrical measurements obtained at 75mm are sufficiently remote from any effects at the concrete surface (e.g. drying, wetting) and will reflect cement hydration, pozzolanic reaction and microstructural development.

The resistivity of the concrete at each electrode-pair was evaluated by multiplying the measured resistance by a calibration factor, denoted k (i.e. equivalent to the geometrical factor A/L in equation (1) above). This was obtained by calibrating with the resistivity of 100mm cubes cast from the same batch as the slabs used in the experimental programme. The measured resistance, R (ohms), across the electrode-pairs were then converted to resistivity, ρ (in Ωcm), through the relationship,

$$\rho = kR \quad \Omega\text{cm} \quad (4)$$

For the electrode-array, k was obtained as 2.41cm.

4.2 Materials

The concrete mixes are presented in Table 1 with a crushed granite aggregate used throughout together with a mid-range water reducer (SikaPlast 15RM). Specimens were cast as 250×250×150mm (thick) slabs in steel formwork with the 250×250mm face cast against the formwork used as the working face. An electrode array described above was positioned at the plan centre of each slab; wiring from the embedded array was ducted out of the slab and terminated with a multipole D-connector. The 28-day and 180-day concrete compressive strengths (f_{28} and f_{180} , respectively) are presented in Table 1.

4.3 Testing Regime and Monitoring

On demoulding at approximately 24-hours, each specimen was wrapped in damp hessian and placed in a heavy-duty polythene bag which was then sealed. The specimens were then left in a laboratory at constant temperature ($20\pm1^\circ\text{C}$) for a period of 27-days. At this time, the four vertical

faces and the as-cast face of the specimens were painted with two coats of a proprietary sealant and exposed to a laboratory environment $20\pm1^{\circ}\text{C}$, $55\%\pm5\%\text{RH}$. After approximately 7-days, all samples were ponded with water for a period of 24-hours. This ensured the surface region of all specimens was in a similar saturated state. Samples were then allowed to dry for further 7-days before being subjected to a cyclic ponding regime comprising 1-day wetting followed by 6-days drying.

Electrical resistance measurements were obtained by connecting the samples to a multiplexing system and an auto-ranging data logger. The resistance of the concrete between each electrode-pair was obtained at a fixed frequency of 1kHz; the signal amplitude was 350mV with a measurement integration time of 1.0 second. Thermistor measurements were also acquired using the same system which were, subsequently, converted to temperature (in $^{\circ}\text{C}$). On demoulding at 24-hours, measurements on the array were then taken on a 12 hour cycle extending over periods extending up to 350 days.

Table 1. Summary of concrete mixes (pl =plasticiser, w/b = water-binder ratio, FA = fly ash, GGBS = ground granulated blast-furnace slag).

Mix Designation	w/b	CEM I kg/m ³	GGBS kg/m ³	FA kg/m ³	20mm kg/m ³	10mm kg/m ³	Fine (<4mm) kg/m ³	pl l/m ³	f_{28} MPa	f_{180} MPa
PC	0.35	378	-	-	787	393	787	5.15	79	88
	0.65	263	-	-	790	395	790	-	39	46
GGBS/65	0.35	132	244	-	782	391	782	5.12	65	76
	0.65	92	170	-	786	393	786	-	31	40
FA/35	0.35	242	-	130	773	386	773	5.06	65	81
	0.65	169	-	91	780	390	780	-	24	38

5.0 RESULTS AND DISCUSSION

Only a selection of results are presented for illustrative purposes, with discussion focussing on the electrodes positioned at 75mm from the concrete surface.

5.1 Preliminaries

Fig. 2 presents the resistivity, ρ , versus time response for the electrodes positioned at 10, 20, 30 and 75mm from the surface for the PC concrete mix (w/b=0.65) with data presented between 100 and 200 days after casting. (Note: for reasons of clarity, only every 5th measurement point is highlighted). The influence of the cyclic wetting/drying regime on the resistivity of the concrete is

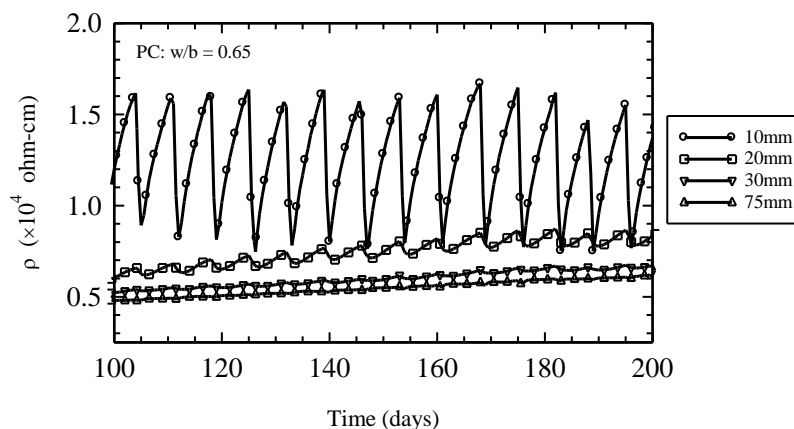


Fig. 2. Variation in resistivity for electrode-pairs at 10, 20, 30 and 75mm from the concrete surface undergoing cyclic wetting (PC concrete, w/b=0.65)

clearly evident for the electrode-pairs positioned 10mm and 20mm from the surface, with periods of wetting resulting in a decrease in resistivity and drying accompanied by an increase in resistivity. The electrode-pair positioned at 75mm, however, displays a continual increase and is not influenced by wetting/drying at the concrete surface. Similar responses were obtained for the other concrete mixes and it is for this reason that measurements from the electrode-pair positioned at 75mm from the exposed surface, and still within the near-surface region, are discussed below.

5.2 Resistivity/Time response

Fig. 3(a) presents the resistivity for concrete mixes with $w/b=0.35$ over the initial 4-weeks after demoulding. The resistivity of all mixes display a continual increase although the influence of the SCM's on resistivity is also evident. For periods <3 days, the PC concrete displays a higher resistivity than the FA/35 and GGBS/65 mixes and reflects the initial slower reaction of these systems. However, for periods >3 -days, the resistivity of the GGBS/65 concrete achieves higher values than the PC concrete whereas for the FA/35 concrete, this effect does not occur until approximately 15 days. Similar trends are observed in Fig. 3(b) for concrete mixes with $w/b=0.65$ although the resistivity of the FA/35 mix only achieves higher values than the PC mix at periods in excess of 28-days. It is also observed that an increase in the w/b ratio results in a decrease in resistivity.

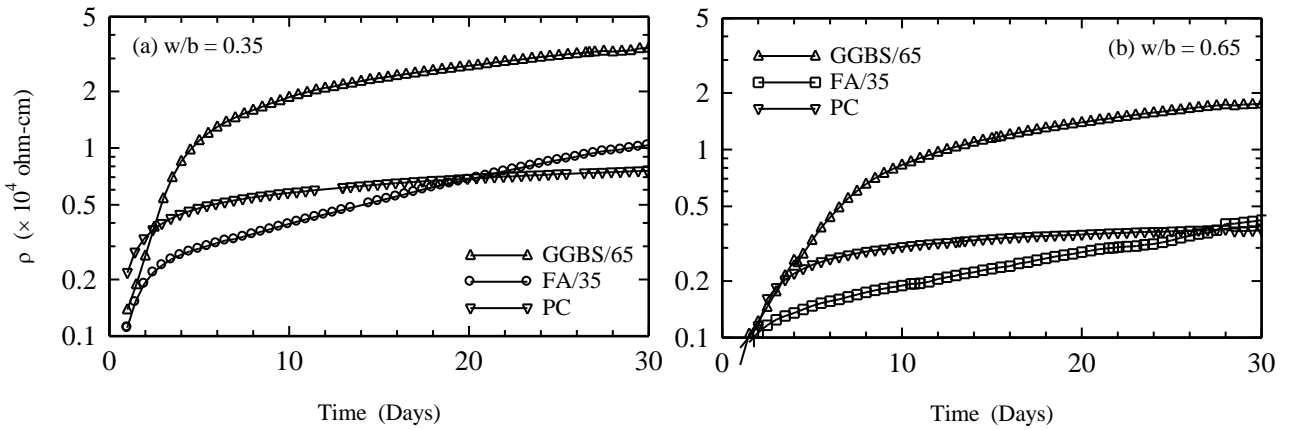


Fig. 3. Increase in resistivity during initial 28-days after casting for (a) $w/b=0.35$ and (b) $w/b=0.65$.

Figs. 4(a) and (b) display the resistivity at 75mm from the surface from 7-days up to approximately 350 days for 0.35 and 0.65 w/b ratios (Note: again, for clarity, only every 20th measurement point is highlighted on these curves). The influence of SCM's on the resistivity is clearly evident from these Figures; at the end of the test period, the resistivity of the FA/35 and GGBS/65 mixes are almost an order of magnitude higher than the PC mix, at both w/b ratios. The increase in resistivity for both the GGBS and FA concretes reflects the on-going pozzolanic activity and pore structure refinement during the post-curing period.

The increase in resistivity for the concretes presented in Fig. 4 can be represented by the equation:

$$\rho = \rho_{ref} \left(\frac{t}{t_{ref}} \right)^n \quad (5)$$

where, ρ is the resistivity at time, t (in days); ρ_{ref} is the resistivity at a reference time, t_{ref} , and n could be regarded as an *aging* exponent which will be related to hydration and pozzolanic reaction. The reference time for the current work is taken as 28-days, hence $t_{\text{ref}} = 28$ -days and the respective resistivity at 28-days, ρ_{ref} , obtained from Fig. 3. Best-fit curves to the data are plotted on Figure 4(a) and (b) (solid lines) through the measurement points with the fitting equations presented on these Figures. It is interesting to note that the aging exponent, n , would appear to be virtually independent of w/b ratio and is binder specific. This could be explained by the fact that the influence in of the w/b ratio is accounted for in the 28-day resistivity, ρ_{ref} .

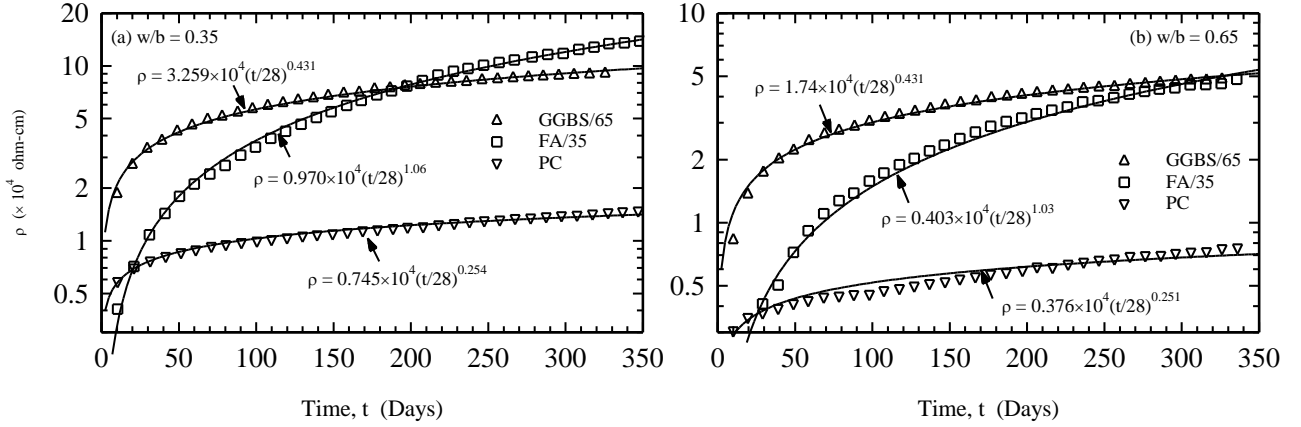


Fig. 4. Increase in resistivity (at 75mm) over the period 7-350 days after casting for (a) $w/b = 0.35$ and (b) $w/b = 0.65$. Curve fits in the form of equation (5) are also presented.

Although the equations on these Figures were developed on the best-fit line to all the data points for a particular w/b ratio (i.e. over 700 measurement points), a curve could be evaluated from fewer measurements, which has obvious practical implications. For example, Figs. 5(a) and (b) present the best-fit curves (solid lines) based on resistivity measurements at 3 measurement points (7, 28 and 56 days) using the same reference time of 28-days. For comparative purposes, the best-fit curves based on all the measurement points on Fig. 4 are also presented on Fig. 5 (dashed lines).

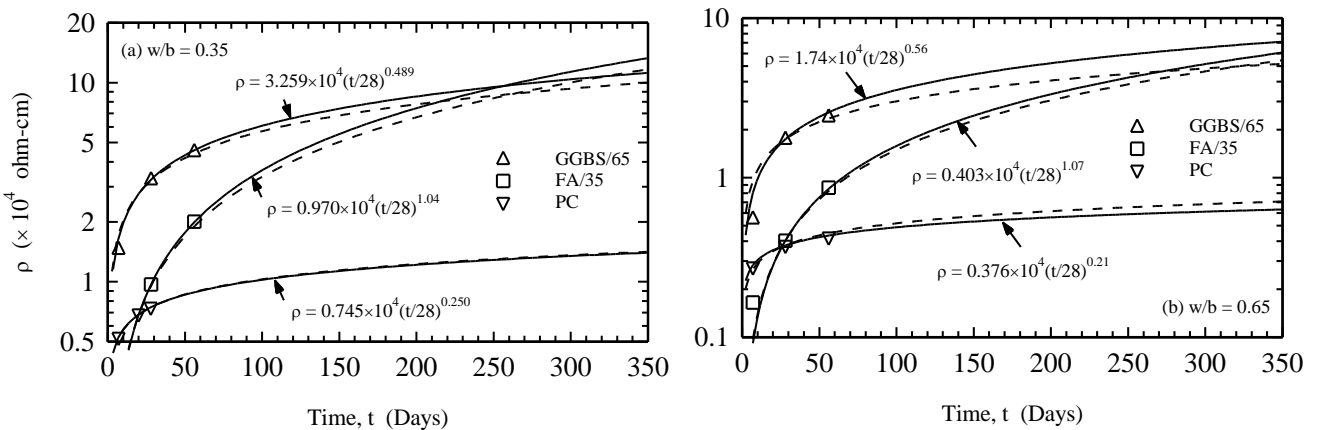


Fig. 5. Curve-fits to resistivity measurements based on 3 (three) measurement points – 7, 28 and 56 days for (a) $w/b = 0.35$ and (b) $w/b = 0.65$.

5.3 Towards a Performance-Based Index

From the definition of formation factor (F) in equation (2) above, and its relationship with diffusivity through equation (3), the concretes could be ranked in terms of their resistance to ionic penetration using the parameter, F . However, the formation factor requires a knowledge of the pore-

water resistivity. A straightforward procedure for estimating pore-water resistivity from the concentrations of Na^+ , K^+ and OH^- ions in the pore-water has been developed [16, 17]. In summary, for a particular degree of hydration, the concentrations of Na^+ and K^+ are computed from the binder composition (Table 2) and assuming that 75% of the Na_2O and K_2O will be released into the pore-water. The concentration of OH^- is then deduced from the electro-neutrality condition. Using the equivalent conductivity of each ion, these data are then used to compute the pore solution electrical resistivity, ρ_p .

Table 2. Oxide analysis of cementitious materials used in experimental programme.

% by weight	PC	FA	GGBS
SiO_2	20.68	51.0	34.33
Al_2O_3	4.83	27.4	12.60
Fe_2O_3	3.17	4.6	0.60
CaO	63.95	3.4	41.64
MgO	2.53	1.4	8.31
TiO_2	+	1.6	+
P_2O_5	+	0.3	+
SO_3	2.80	0.7	+
K_2O	0.54	1.0	0.47
Na_2O	0.08	0.2	0.25

Table 3. Formation factors (F) for the concrete mixes in Table 1 at 28-days (F_{28}) and 56-days (F_{56}).

Mix	w/b	ρ_p (Ωcm)	ρ_{28} ($\times 10^4 \Omega\text{cm}$)	ρ_{56} ($\times 10^4 \Omega\text{cm}$)	F_{28}	F_{56}
PC	0.35	10.7	0.725	0.865	677	808
	0.65	23.8	0.368	0.445	154	186
GGBS/65	0.35	27.9	3.23	4.46	1157	1598
	0.65	63.7	1.69	2.32	265	367
FA/35	0.35	8.31	0.952	2.01	1146	2418
	0.65	18.2	0.394	0.842	216	462

Table 3 presents the computed F values for each concrete mix in Table 1 at both 28-days (F_{28}) and 56-days (F_{56}); in calculating ρ_p , an average degree of hydration of 80% has been assumed throughout for each binder type. With reference to Table 3, decreasing the w/b ratio from 0.65 to 0.35 results in a fivefold (approximately) increase in F for each mix. Moreover, the beneficial effect of the FA and GGBS is clearly evident, particularly at longer time-scales; although these materials may not necessarily create concretes of lower porosity than their PC counterparts, it is of a more disconnected and tortuous in nature. It is interesting to note that whilst the GGBS/65 concrete displays the highest resistivity, when the resistivity of the pore-water is considered, it is out-ranked by the FA/35 concrete in the longer-term. From equation (3) above, the *instantaneous* diffusion coefficient, D, of a particular ionic species (e.g. Cl^-) could also be approximated from F (i.e. $D \approx D_0/F$) and the free-solution diffusion coefficient of that ion, D_0 (e.g. D_0 for the chloride ion is $1.84 \times 10^{-9} \text{ m}^2/\text{s}$ [19]).

6. CONCLUDING COMMENT

The work detailed has presented developments in the application of electrical resistivity measurements as a potential test method to rank the relative performance of a range of concrete

mixes. The measurements were used in combination with the pore-fluid resistivity, the latter evaluated from the ionic concentration in the pore water. These values were then used to assess concrete performance in terms of Formation Factor (F). In general terms, the higher the F value for the concrete, the better its performance rating; however, work is still required to obtain values, or range of values, for F for concrete classification purposes.

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